9.7 APPENDIX - SOILS REPORT

LIMITED SOIL INVESTIGATION
for
THE CASTRO ADOBE
SANTA CRUZ COUNTY, CALIFORNIA

for
MR. DAVID POTTER
c/o GILBERT ARNOLD SANCHEZ, INC.
Santa Cruz, California

BY
JAMES C. REYNOLDS & ASSOCIATES, INC.
GEOTECHNICAL ENGINEERS
85123-S76-H2
December 1985

RECEIVED
DATE 1/2/86
BY
GILBERT ARNOLD SANCHEZ
ARCHITECTS, INCORPORATED

JAMES C. REYNOLDS & ASSOCIATES, INC.
Geotechnical Engineers

85123-S76-H2
24 December 1985

Mr. David Potter
c/o Gilbert Arnold Sanchez, Inc.
3022 Glen Canyon Road
Santa Cruz, CA 95060

Subject: Castro Adobe, 184 Old Adobe Road
Watsonville, California

Dear Mr. Potter:

In accordance with the request of your Architect, Mr. Sanchez, we have performed a limited soil investigation at the west end of the subject residence, located on Adobe Road, about four miles northwest of Watsonville, California.

The accompanying report presents our conculsions and recommendation as well as the results of the soil investigation on which they are based. If you have any questions concerning the data, conclusions or recommendations presented in this report, please call our office.

Our conclusions and recommendations are based upon six feet deep hand auger test boirngs. Other test boirngs in the vicinity indicate increasing quality of soil with increased depth. Our recommendations are based on the assumption that this is true for this site, however, this cannot be guarenteed unless deeper borings are made. This will require additional expense, plus some tree trimming.

Purpose and Scope

This report describes the soil investigation and presents results, including recommendations, for the underpinning of the west wall of the Adobe Residence. The purpose of this study was to determine the subsurface soil condiitons, to evaluate the cause of the present subsidence, and then to present recommendations for new underpinning foundation. Recommendations for general improvements are also presented.

Location and Description

The project site is located on the east side of Old Adobe Road at No. 184, and about four miles northwest of Watsonville in an unincorporated portion

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Santa Cruz County. The site is at the crest of a north-south trending ridge. At the time of our field investigation the west end of the two-story adobe building had evidenced measurable subsidence. This subsidence has also extended down the north and south exterior walls some ten to twelve feet. A recent excavation along the west wall indicates that the west wall foundation consists of a place stone footing, which extends about eight inches below the ground surface. Three counterfort type walls have been placed perpendicular to the west wall, see Figure No. 1, "Site Plan Showing Test Borings."

The crest of the ridge appears to have been levelled to provide a level building pad area for the adobe building. The original crest was relatively level, however beyond the building pad area the ground slopes off at a moderate gradient.

These details are based on a site visit by the Geotechnical Engineer in the company of your Architect, Mr. Gilbert Sanchez, and your Structural Engineer, Mr. Ken Yuen, and on plans provided by the Architect.

Field Investigation

The field investigation was conducted on 6 December 1985, with the drilling of two test borings at the approximate locations shown on Figure No. 1, "Site Plan Showing Test Borings."

The test borings were drilled to depths of six feet below the existing ground surface and were advanced by using a hand operated auger. The soils encountered were logged continuously in the field during the drilling operations. Soil samples were obtained by driving a 2.0 inch O.D. thin walled Shelby sample into the ground at various depths. The sampler was dynamically driven with a 10# sledge hammer. Figure No. 2, Appendix A, "Logs of Test Borings", show a graphic presentation of the soil profiles as exposed in the test borings and the locations and depths at which the soil samples were obtained. The stratification lines prepresent the approximate boundaries between material types as the actual transition may be gradual. The field penetration values are shown on the logs opposite the depth at which the sample was obtained.

Laboratory Testing

The laboratory testing program was directed towards a determination of the physical and engineering properties of the soil underlying the site. Moisture content and dry density tests were performed on representative soil samples to determine the density of the soil and the moisture variations. A direct shear test was conducted on a sample of the potential foundation soil.

The results of all laboratory testing appear on the "Logs of Test Borings", Appendix A, opposite the sample tested.

Soil Conditions

The native soils encountered in the test borings to the depths penetrated show uniformity variations across the site. The upper three and a half feet of native soil consist of loose clayey sand materials, which grades to a medium dense to dense sand with some clay binder. These soils are indicative of the Aromas Formation generally encountered in this area. The Aromas formation extends to depths in excess of one hundred feet, and generally increase in density and strength parameters with increased depth. No free ground water was encountered.

Faults and Seismicity

Santa Cruz County is a seismically active region and has several active faults running through it. The nearest known major active land fault is the San Andreas Fault, a major crustal break, located about 4½ miles to the northeast, which has been traced from the Gulf of California north to Point Arena. The San Andreas Rift Zone includes many small subparallel faults along which varying amounts of predominately horizontal movements have been distributed over millions of years. Movement along the San Andreas Fault was responsible for the large earthquake which occurred on 18 April 1906. This earthquake measured 8.3 on the Richter Scale and caused some damage to man-made structures in the Monterey Bay Area. Hall, et. al (1974), states that the San Andreas Fault has a high potential for surface rupture with a recurrence interval of 50 to 100 years. It has been estimated that the maximum likely earthquake in a 50-year period along the San Andreas Fault will have a magnitude of 7.0 to 8.9 (Richter).

Another fault in the general area is the Zayante Fault, located two miles northeast of the site. This fault is associated with the San Andreas and is considered at least potentially active by the State of California. The Zayante Fault is considered to have a moderate potential for surface rupture. The San Gregorio Fault and the Monterey Bay Fault Complex are located off shore.

Seismic hazards to man-made structures include rupture, ground shaking, landsliding, liquefaction, lurch cracking, and differential compaction. Ground shaking is considered the only seismic hazard which may affect the structure built on the site.

Surface rupture usually occurs along lines of previous faulting. Since there is no evidence of active faulting in the immediate vicinity of the site and the nearest known potentially active fault (the Zayante Fault) is located about two miles from the site, the chances of surface rupture across the site is remote.

Ground shaking caused by earthquakes is a complex phenomenon. Structural damage can result from the transmission of earthquake vibrations from the ground into the structure. The intensity of shaking depends on, amongst other variables, the proximity of the site to the focal point of the earthquake. Structures build on unconsoilidated materials generally experience movements of higher amplitude and lower acceleration. In the event of an earthquake, frame and semi-rigid structures with proper seismic parameters incorporated into the design and construction should display only minimal damage.

Landsliding is common during large magnitude earthquakes. Since the area where the structures will be located has only a moderate gradient and with the recommendations included in this report, the potential for slope failure is minimal.

Liquefaction, lurch cracking, and differentail compaction tend to occur in loose, unconsolidated soils. The liquefaction potentials for the Quarternary deposits in Santa Cruz have been estimated by Durpe (1975). The report indicated that this site has a low potential for liquefaction. The results of our soil investigation, based on the soil consistency, the location of the ground water table and the general nature of the subsurface soils, indicate that the potential for liquefaction to occur within the limits of this site is nil.

DISCUSSIONS, CONCLUSIONS AND RECOMMENDATIONS

General

1. The results of our investigation indicate that from a soil engineering standpoint the existing adobe structure may be underpinned along the western side with designed foundations.

2. The condition of the site and our discussions indicate that no grading will be required.

Foundations

3. The original stone formation and the subsequent three counterfort footings are all founded within the upper stratum of loose clayey sand material; the original footings have about three feet of loose soil beneath them while the counterfort wall footings are resting on at least eighteen inches of loose material. These footings were destined to undergo eventual subsidence, particulary following the two very wet winters of 1982 and 1983.

4. Because of the loose surface soil stratum, we recommend that the western wall of the adobe residence be underpinned, and supported by reinforced drilled cast-in-place concrete piers and grade beam.

5. The piers should be designed as skin friction piers, with an allowable side wall bearing value of 350 p.s.f. for dead plus live load. This value may be increased by one-third to include the effect of short term wind and seismic type loadings. The upper four feet (4') should be neglected for determining allowable bearing depth of penetration. The minimum pier diameter should be twelve inches (12").

6. The piers and grade beams should contain steel reinforcement as directed by the Project Structural Engineer in accordance with applicable UBC or ACI Standards.

Drainage, Surface

7. Surface water must not be allowed to pond adjacent to the building foundations nor on any part of the building pad.

8. Continuous roof gutters are recommended. The outlets from the gutter downspouts should be provided with adequate capacity to carry storm water from the structure to a suitable discharge point to reduce the possibility of soil saturation and erosion.

9. Final grade should be provided with a positive gradient away from all foundations in order to provide rapid removal of the surface water to an adequate discharge point. Accumulations of runoff should be handled by closed pipes, lined ditches, catch basins or similar type facilities.

10. The buildings and surface drainage facilities should not be altered, nor any filling or excavation work performed after the initial construction without consulting a Geotechnical Engineer.

11. Irrigation activities at the site should be done in a controlled and reasonable fashion.

Plan Review

12. We respectfully request an opportunity to review the plans during preparation and before bidding to insure that the recommendations of this report have been included and to provide additional recommendations, if needed.

If you have any questions regarding these recommendations or any additional data you may need, please contact this office at your convenience.

Very truly yours,

JAMES C. REYNOLDS & ASSOCIATES, INC.

James C. Reynolds

James C. Reynolds
C.E. 15285

JCR:sr
Copies: 3 to Mr. David Potter, c/o Gilbert Sanchez, Inc.
1 to Ken Yuen, Structural Engineer

85123-S76-H2
24 December 1985

LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. The recommendations of this report are based upon the assumption that the soil conditions do not deviate from those disclosed in the borings. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that planned at the time, our firm should be notified so that supplemental recommendations can be given.

2. This report is issued with the understanding that it is the responsibility of the owner, or his representative, to ensure that the information and recommendations contained herein are called to the attention of the Architects and Engineers for the project and incorporated into the plans, and that the necessary steps are taken to ensure that the Contractors and Subcontractors carry out such recommendations in the field.

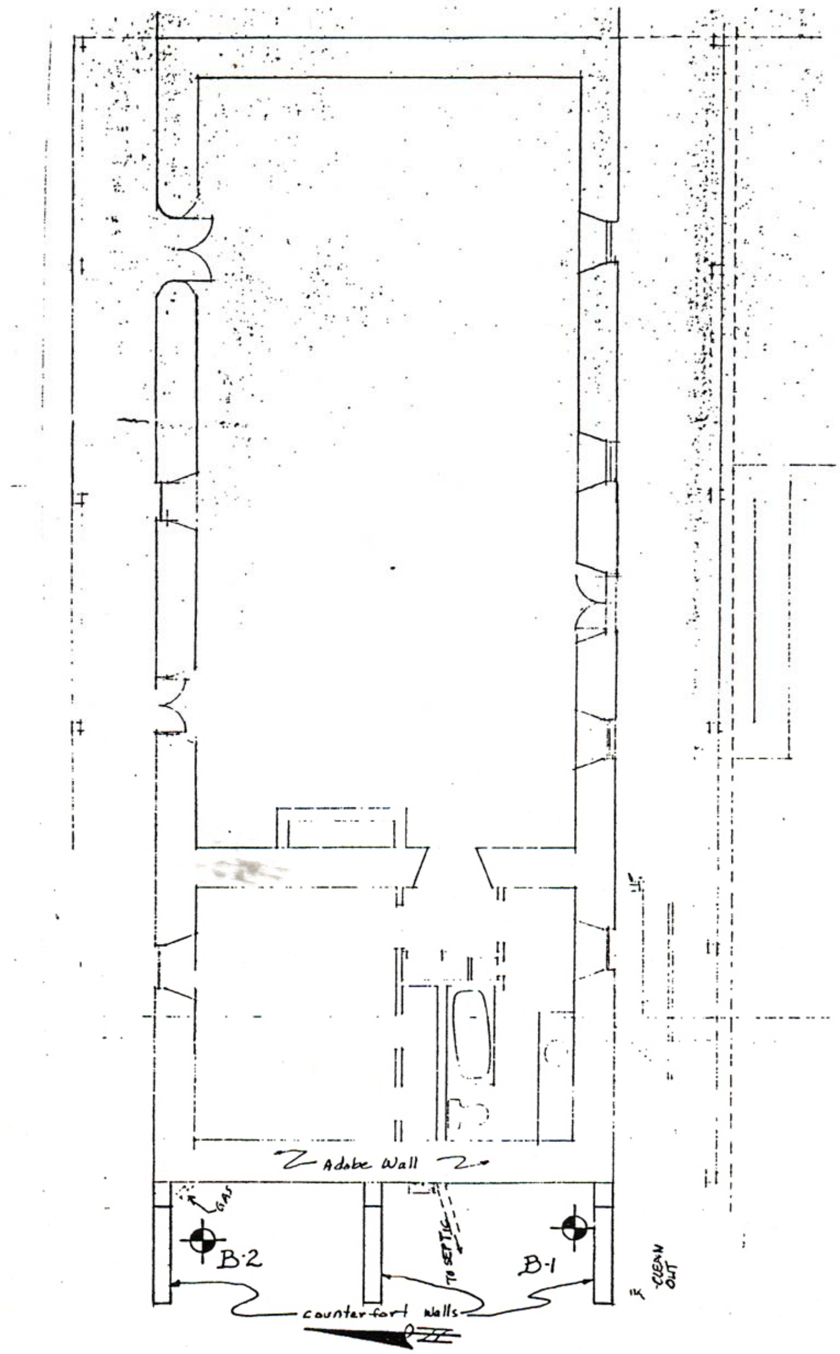
3. The findings of this report are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural processes or to the works of man, on this or adjacent properties. In addition, changes in applicable or appropriate standards occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated, wholly or partially, by changes outside of our control. This report should therefore be reviewed in light of future planned construction and the then current applicable codes.

4. <u>EXCLUSION OF WARRANTIES</u>: Our services are to consist of professional opinion only. NO WARRANTY, EXPRESS OR IMPLIED, INCLUDING ANY IMPLIED WARRANTY OF MERCHANTABILITY OF FITNESS FOR THE PURPOSE is made or intended in connection with our work or by the proposal for consulting or other services or by the furnishing of oral or written reports or findings. If the owner (client) desires assurances against project failures, owner agrees to obtain appropriate insurance through his own insurance broker, which insurance shall include a waiver of subrogation clause as to James C. Reynolds & Associates.

APPENDIX A

Site Plan Showing Test Boring

Log of Test Borings



JAMES C. REYNOLDS, & ASSOCIATES, INC. | **FIGURE NO.** 1-Site Plan Showing Test Borings

7

PM30283/B

LOGGED BY JCR DATE DRILLED 14 Dec. 1985 BORING DIAMETER 6" BORING NO. 1&2

Depth, ft.	Sample No. and type	Symbol	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft-lbs.	Qu - t. s. f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
			BORING NO. 1						
1–3	1-1* S		Medium brown Clayey SAND, moist, loose Damp	SC			84	13.3	
3–6	1-2 S		Reddish-brown SAND, with clay binder, moist, loose, medium dense at 3.5 feet Dense at 4 feet, damp Very hard to hand auger	SW			117	9.8	Direct Shear C=400 p.s.f. Ø=32°
6			Boring terminated at 6 feet *Note: Sample obtained directly from beneath the counterfort footing.						
			BORING NO. 2						
0–3	2-1 S		Medium brown Clayey SAND, moist, loose Damp	SW			96	14.0	
3–6			Reddish-brown SAND with clay binder, medium dense Damp Very hard to hand auger	SW					
6			Boring terminated at 6 feet						

JAMES C. REYNOLDS, & ASSOCIATES, INC. FIGURE NO. 2- Log of Test Boring

PM30283/A

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(415) 968-9351

JOB NO. 85M-26
PROJECT CASTRO ADOBE
DATE JAN. 8, 86. BY K.S

STRUCTURAL COMPUTATIONS

— for —

THE CASTRO ADOBE

SANTA CRUZ COUNTY, CALIF.

REGISTERED PROFESSIONAL ENGINEER
KENNETH H. C. YUEN
No. 1755
STRUCTURAL
STATE OF CALIFORNIA

ARCHITECT

GILBERT ARNOLD SANCHEZ
SANTA CRUZ.

SHEET 1 OF 4

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JOB NO. 85M-26
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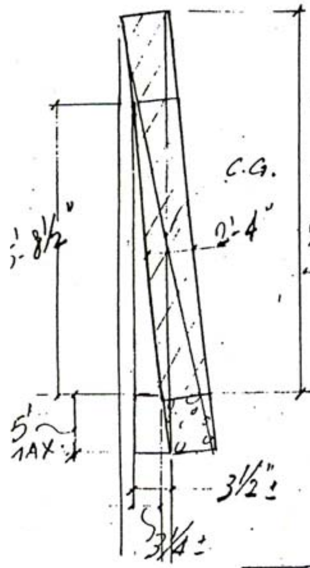
DATE JAN. 7, 85 BY K.S.



(A) CHECK EXISTING CONDITION.

ADOBE WT. = 120 pcf

TAKE 1'-0" WIDE @ MAX. HEIGHT ADOBE WALL TO ANALYSE (VERTICALLY)

$e \leq 3''$ MAX. FROM C.G. OF 1'-0" WIDE ADOBE WALL TO ℄ EXIST. FTG.

$e_{MIN} > 1.5''$

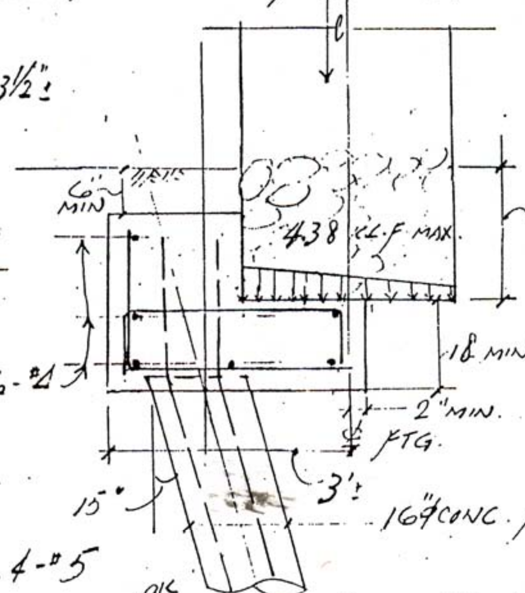
P = TOTAL WT. OF 1'-0" WIDE ADOBE WALL $= (12+16) \times 1.5 + 120\ pcf \times 1' \times 21.5' \times 2.33 = 6.1^k$

$P_{SOIL\,(MAX.)} = \frac{6.1^k}{2.33'} \pm \frac{6.1^k \times 0.25 \times 1.17'}{\frac{1}{12} \times 1 \times 2.33^3}$

$= 2.62\ k.s.f \pm 1.70\ k.s.f.$

$= 4.31\ K.S.F$

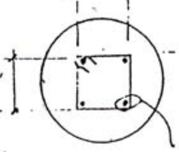
TOO BIG
MUST ADD CONC. GRADE BM & CONC. PIERS



) CHECK (N) CONC. PIER

$P_{SOIL\,(MIN.)} = \frac{4.9^k}{2.33'} \pm \frac{4.9^k \times .17 \times 1.17'}{\frac{1}{12} \times 1 \times 2.33^3}$

$= 2.10 \pm 1.03 \approx 3.1\ K.S.$



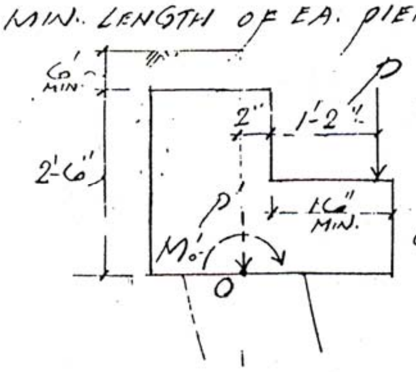
16" ϕ CONC. PIER. $G.F = 2\pi(.67') \times 350\ P.S.F = 1.47^{k/}$

TOTAL WT. ON PIER $= \left(150 \times 2' \times 3'_{AVE} + 6.1^k + 120\ pcf \times 1' \times \frac{17.2' \times 2.33}{2} + (12+16) \times 1.5\right) \times 26.5'$ MAX.
$= 170^k$

(.9k above first term; 4.9k above last term)

ACTUALLY 22' ONLY.

MIN. LENGTH OF EA. PIER = 170^k @ 6 PIER × 1.47 k/ = 19.2 EFFECTIVE LENGTH.



$P_{AVE} = 5.5^{k/}$

$M_0 = 5.5^{k/} \times 1'\text{-}4''_{MAX} = 7.3^{k'/}$

MIN. BRG. ON CONC
$= 3''_{MIN} \times 12'' \times 625\ psi_{MIN.}$
$= 21.6^{k/} >> 5.5^{k/}$ o.k.

CHECK LENGTH OF CONC. PIER DUE TO M'

$L = \sqrt[3]{\frac{14.53 \times 7.3^{k'/} \times 5.0'}{2 \times 0.25_{MIN.} \times 1.33'}} = 9.3' << 19.2'$ o.K

o.K

SHEET 2 OF 4

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JOB NO. 85M-26
PROJECT CASTRO ADOBE
DATE JAN. 7, 85 BY K.S.

MAX. MOMENT @ EA. PIER = 11^K x 9.3' x 2/3 = 68.2^K' @ 1.44 x 13" = 3.6 □"

MAX. SHEAR @ EA. PIER = 250 P.S.F x 9.3 x 9.3 x 1/2 = 11^K

MIN. PASSIVE EARTH PRESSURE

4-#5 O.F. = 3.0 x 4 = 11^K

CHECK. MIN. CONC. SHEAR (W/O. REINF.) = π (8")² x 100 PSI MIN. = 20^K > 12.5^K O.K

CONSIDER BOTH O.K

CHECK BOND STRESS OF #5 INTO CONC GRADE BM.

MIN. BOND LENGTH. 40 DIA. = 30"

Mo' = 7.3^K'/' x 5' = 37^K' @ 10 1/2" = 44^K @ 20^KSI = 2.2 □"

EA. #5 BOND FORCE = 300 PSI MIN. x 1.963" x 30" = 17.7^K << 44

4.8√f'c / D

NOT WORK SEE SH. 4 OF 4 FOR ADDED TENSION PIER

2) CONC. GRADE BM.

5.5^K/' 5'

M MAX = 1/8 x 5.5^K/' x 5'² = 17.2^K'/' @ 1.44 x 20 MIN. = .60 □"

(LONGIT. DIR)

M TRANSV. BAR = 5.5^K x 1.17 = 6.4^K'/' @ 1.44 x 16 MIN. = .31 □"/' 3-#4 BOT.

← #5 @ 12

CHECK. SOIL PRESSURE ~~IF~~ W/O. CONC. PIER. (WITH GRADE BM ONLY)

$$P_{SOIL\ (MAX)} = \frac{5.5^K/'}{3'} \pm \frac{7.3^{K'/'} \times 1.7'}{\frac{1}{12}\times 1 \times 3^3} = 1.83\ K.S.F + 5.4\ K.S.F = 7.2\ K.S.F.$$

MUST USE CONC. PIER ANYHOW

MAX. SHEAR @ CONC. GRADE BM

SHEAR FROM VERTICAL LOAD = 5.5^K/' x 3.67' CLR = 20^K (ALL) @ 24" = 46 PSI

M_t IN LB-IN.

SHEAR FROM TORSION MOMENT = $\frac{5 \times [(7.3^{K/'} \times 5') \times 1000 \div 12]}{18"_{MIN}{}^2 \times 24"_{MIN.}}$ = 2.22 P.S.I

ALLOWABLE TORSIONAL SHEAR FOR CONC. BM $V_c = 1.1\sqrt{f'c}$ = 60 PSI >> 2.22 PSI O.K.

USE NOM REINF.

1'-4"±

CHECK. C.G. OF GRADE BM

$$C.G._{\bar{Y}} = \frac{1.5' \times 2.5 \times 0.75' + 1.5' \times 1.5' \times 2.25'}{1.5 \times 2.5 + 1.5 \times 1.5} = \frac{7.875}{6} = 1.3125'$$

$C.G._{\bar{X}}$

SHEET 3 OF 4

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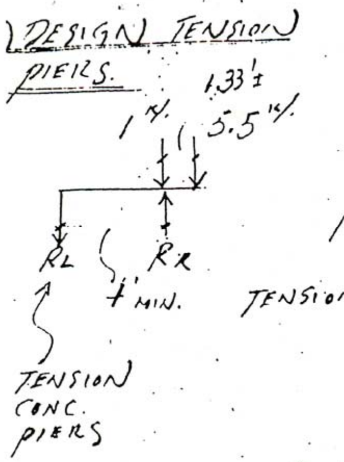
JOB NO. 85-M-26
PROJECT CASTRO ADOBE
DATE JAN 8, 85 BY K.S.

1. DESIGN TENSION PIERS.



$R_L = 5.5^{k/'} \times 1.33 \times 26.5'$ MAX. @ 5' = 39^{k} @ 3 PER = 13^{k} PER PIER

TENSION PIER LENGTH = $9'$ MIN. $\times 1.47^{k/'} > 14^{k}$ O.K.

13.0^{k} @ 20^{ksi} = $.65^{\square''}$ @ 4 BARS = $.16^{\square''}$

4-#5 O.K.

EA. #5 BOND FORCE = 300 psi × 1.963 × 30" = 17.7^{k} O.K.

CHECK CONC. BM.

$13.0^{k} \times 4' = 52.0^{k'}$ ω 1.44 × 24" MIN. = $1.5^{\square''}$

4-#6 OK. G.F. = 4 × .44 = $1.76^{\square''} > 1.5^{\square''}$ O.K.

$M_{WT. OF BM} = (150^{\#} \times 2' \times 2.5') \times 5^2 \times 1/8 = 2.34^{k'}$ @ 1.44 × 30" = $0.05^{\square''}$

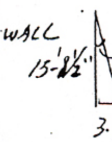
NOM REINF.

NOTE: CAL SH. 2 & 3 DESIGN WITHOUT TENSION CONC. PIER. EVERYTHING O.K. EXCEPT TENSION & BOND OF 1#5 IN CONC. PIER NOT WORK. TENSION CONC. PIER MUST BE ADDED.
CAL. SH. 4 FOR TENSION PIER ADDED.

(E) CHECK MTL STRAP TIE TO ANCHOR INTO LONGIT. WALLS.

F, 1°, $5.5^{k/'}$ AVE.

$F = 5.5^{k/'} \cos 89° = .96^{k/'} \times 26.5 \div 2$
$= 1.5^{k}$ @ EA END WALL

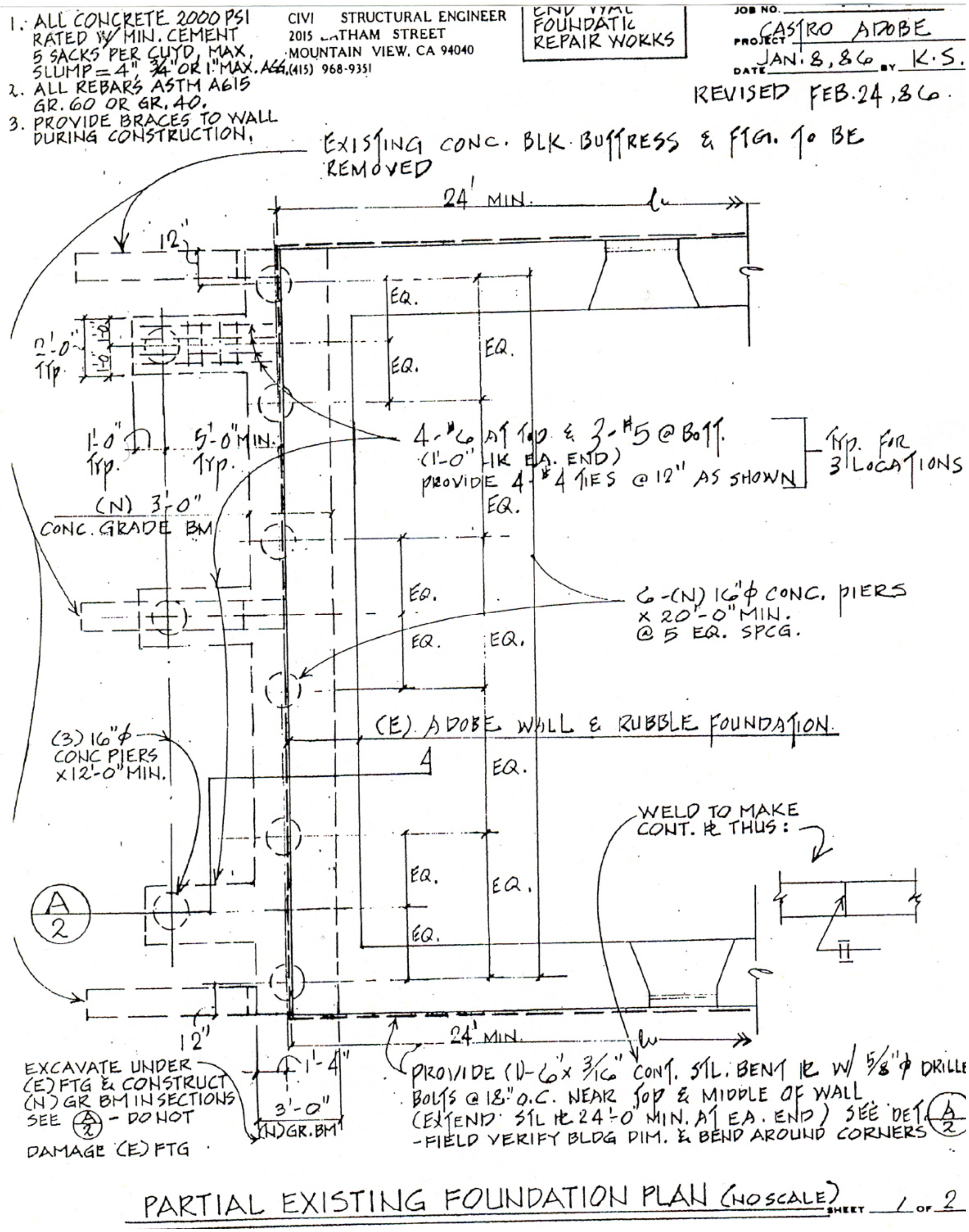


$\theta = \tan^{-1} \frac{3.25"}{188.5"} = 1°$

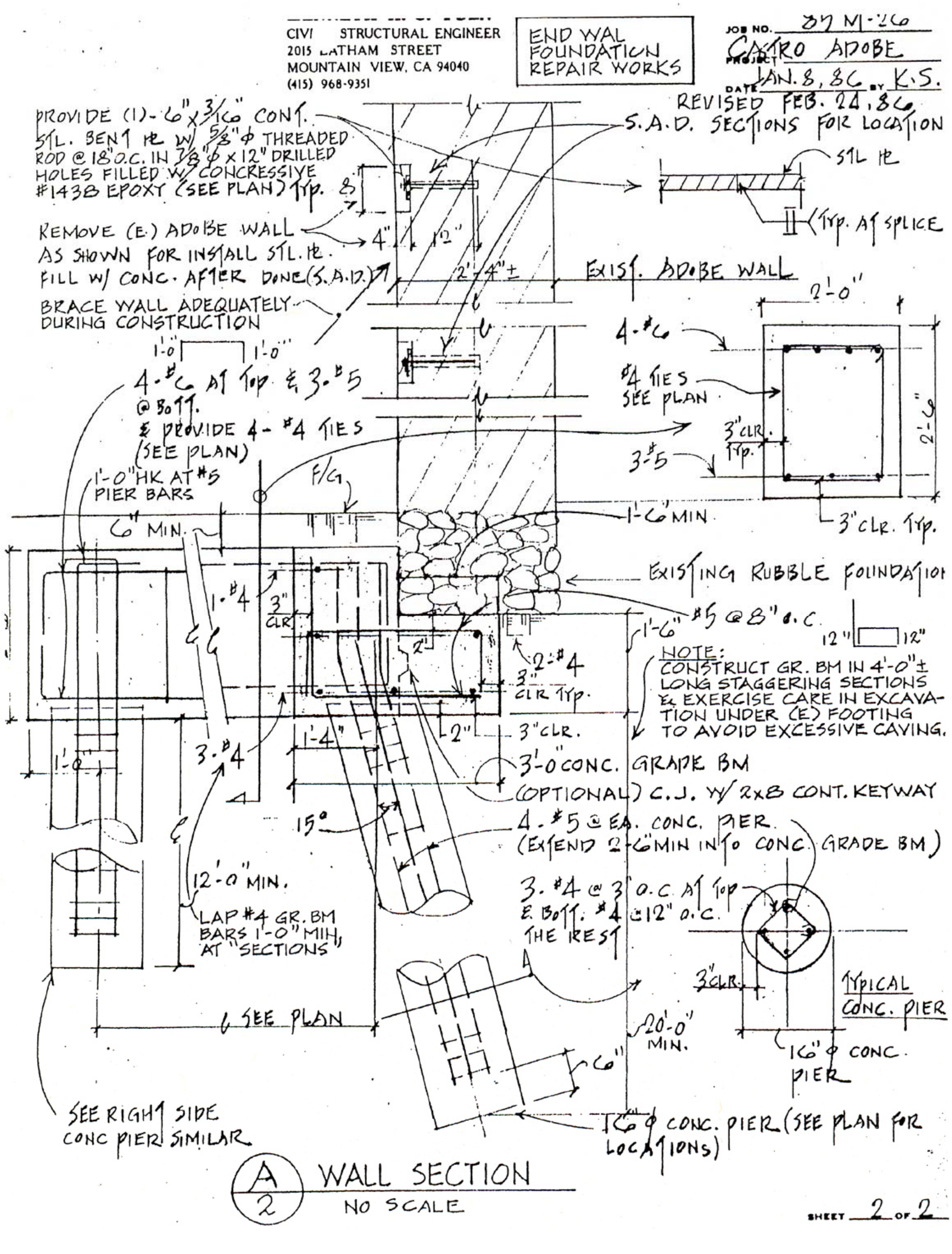
4" × 3/16" CONT. BENT MTL STRAP.
w/ 5/8"φ BOLTS × 12" DEEP. G.F. = $200^{\#}$ EA.

USE MIN. 8 BOLTS G.F. = $8 \times .2^{k} = 1.6^{k} > 1.5^{k}$ O.K.

SHEET 4 OF 4



PARTIAL EXISTING FOUNDATION PLAN (NO SCALE) SHEET 1 OF 2



A/2 WALL SECTION
NO SCALE

SHEET 2 OF 2